EVALUATION OF OLIVE VIEW HOSPITAL BEHAVIOR ON EARTHQUAKE RESISTANT DESIGN

by

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SYNOPSIS - Results from field, capacity and structural dynamics studies on the Olive View Hospital Main Building are correlated and discussed. The relationships of these findings to design are discussed.

INTRODUCTION - The Olive View Hospital main building, a six story reinforced concrete structure was heavily damaged in the two lower stories during the San Fernando Earthquake. The damage was remarkable to observe because of the striking 13.5 in. to 30 in. drift which resulted in the first story. The contest waged by the building during the earthquake was an event of major importance to structural engineers because this recently completed earthquake resistant building was laterally loaded and deformed to a near-collapse condition. Reports and papers on main building studies(1-4) with a variety of objectives have been prepared. The objective of the present paper is to compare the findings of field(5), capacity(5), and structural dynamic modeling studies which have been conducted.

DAMAGES - Investigating the ground floor, the first through fifth floors, and the roof(Fig.1) the following could be observed: 1) the ground story was framed with single story tied columns, multistory square spiral columns, and beam and slab-drop panel systems; a one story retaining wall, separated at the top from the first floor slab by 4in., restrained motion of the building on the north and west sides; hinging occurred at the base of the spiral columns and in the first floor beams and drop panels above; a number of the drop panels punched due to moment; all tied columns on this floor failed in shear; intrastory drift was 6 in. north and 3 in. east; the north retaining wall was pushed 2 in. north at the top and 1 in. north at the ground floor level; 2) in the first story which was framed with multistory square spiral columns and beam and slab-drop panel systems hinging occurred at the top and bottom of the spiral columns; the drift was not uniform due to intrastory rotation; the components of drift were 27 in. north and 13 in. east on the northwest and 13 in. north and 3 in. east on the southeast corners of the building; 3) in the second story,

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which was framed with shear walls and columns, the multistory columns from below changed in cross sectional property to become tied channel columns on the exterior, boundary columns for the shear walls, or double spiral rectangular columns on the interior; beam framing was used on the exterior while slab-drop panel systems were used on the interior; damage was confined to heavy X cracking between floors on the interior north-south shear walls and heavy shear cracking in the exterior channel columns; the cracks usually ran from the top on the north to the bottom on the south except near the north stair tower where X cracking occurred in the channel columns; intrastory drifts were 1 in. or less; 4) in stories three through five the framing configuration and element sizes remained essentially unchanged; shear wall thicknesses were trimmed from 12 in. on the second to 8 in. on the fifth floor and reinforcement ratios were decreased in the higher floors; damages decreased with increasing floor level; in any particular floor the heaviest damage occurred in the channel columns running along north-south lines; 5) located on the roof and covering a portion of each wing was a penthouse composed of walls with columns and beam portal frames going across in the short direction of each penthouse; damage was limited to X cracking in spandrel walls which framed between shear walls coming up from below.

CAPACITIES - The story shear capacity coefficient and lateral stiffness distributions in the building were strongly influenced by transition from frame to shear wall-frame system and concentrations of mass at the roof and first floor. Based on design material properties and ACI(6) capacities the story shear capacity coefficients by floor were: ground 0.32, first 0.44, second 0.32, third 0.39, fourth 0.44, fifth 0.70. The increase in capacity coefficient from ground to the first story occurred because the ground floor columns were unable to hinge at the top before hinging occurred in the beams and at the bottom of the first floor columns. Only a fraction of the moment capacity was developed at the top of the ground floor columns; as a result the ground story shear capacity coefficient was diminished. Based on uncracked sections the ratio of story shears to drifts (stiffnesses) given by floor were; ground 250,000 kips/ft, first 230,000 kips/ft, second 6,000,000 kips/ft, third 4,700,000 kips/ft, fourth 3,800,000 kips/ft, and fifth 2,700,000 kips/ft. The tremendous jump in stiffness is due to shear walls on the second floor.

STRUCTURAL DYNAMICS MODELING - Using initial stiffnesses a normal mode analysis was performed; a 420 coordinate three dimensional idealization was used. The shape characteristics, ϕ, and periods, T, of the six lowest modes were: ϕ_1, first east-west translation, T_1=0.71 sec.(Fig.1); ϕ_2, second north-south translation, T_2=0.70 sec.; ϕ_3, first rotation, T_3=0.53 sec.; ϕ_4, second east-west translation, T_4=0.21 sec. (Fig.1); ϕ_5, second north-south translation, T_5=0.19 sec.; and ϕ_6, second rotation, T_6=0.15 sec.

In order to perform three dimensional modeling up to displacements achieved in the field realistic force-deflection relations for beams and columns were required. For ductile reinforced concrete elements, the moment-curvature models of Ref.(7) were generalized to include biaxial behavior. Column and beam incremental stiffness elements with 12 degrees of freedom (Fig.2) were then devised by integrating the coupled incremental biaxial moment-curvature and axial load-axial shortening relations over the length of the element. To check the element idealization the spiral column test performed at Kajima Institute(8) was modeled as a 48 degree of freedom system. Measured and computed results agree closely(Fig.2). Further refinements for modeling of buckling corner bars which were outside the core and spalling of the shell were added. Tied column failures were modeled using a biaxial shear failure law. In the upper floors similar laws were used except in shear walls where pre- and post-cracking capacities were recognized. Story shear drift relations(Fig.3) demonstrate the effects of various types of framing, hinging, tied column failures, and shear wall stiffness. The ground and first story shear-drift results imply that the large drifts in the first floor were caused by the retaining wall.

Using a coordinate reduction procedure a three dimensional building response history was computed for the Pacoima Dam(1) accelerogram(Fig.4). A 0.4ft. north drift was found in the ground floor, and a 0.2ft. north drift was obtained for the first floor. This result supports the argument that the impact on the retaining wall caused the large first story drift. Another important implication of this result is that ground shaking equivalent or greater than at Pacoima Dam occurred at the site.

CONCLUSIONS - Results from damage, capacity and nonlinear structural dynamics studies agree closely. The nonlinear model which appears to be quite accurate would have predicted a concentration of damage in the lower floors.

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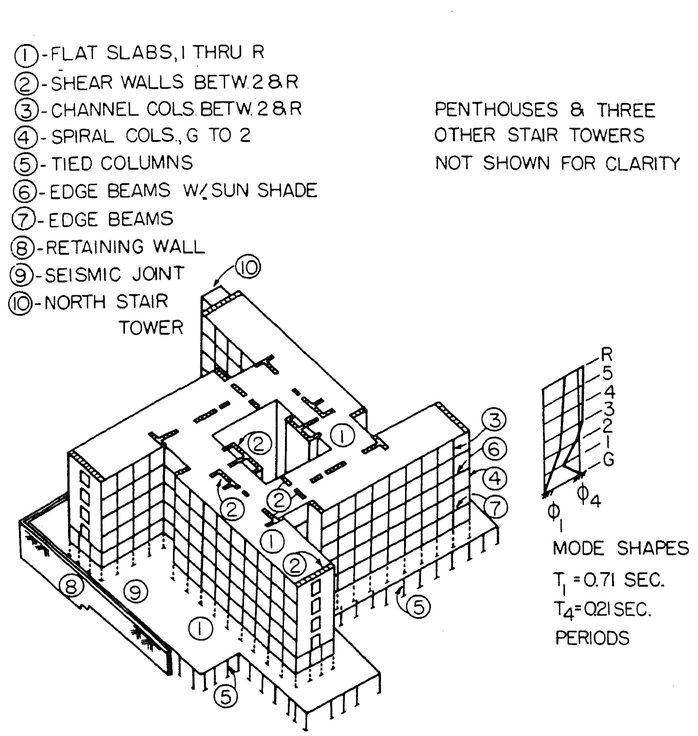

FIG. 1 STRUCTURAL FRAMING & MODE SHAPES OF OLIVE VIEW HOSPITAL MAIN BUILDING.

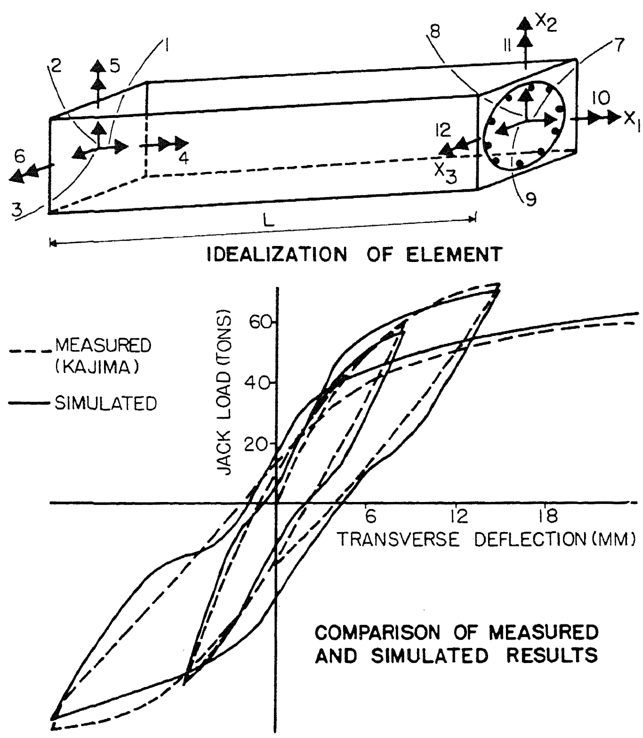

FIG. 2 REINFORCED CONCRETE COLUMN STIFFNESS ELEMENT

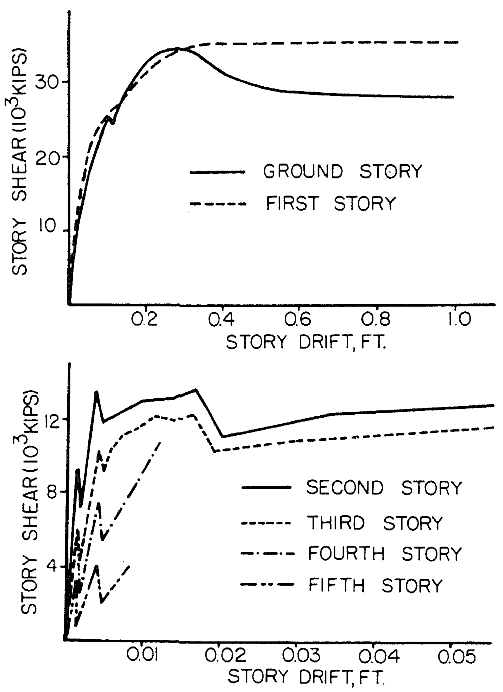

FIG. 3 STORY SHEAR-DRIFT RELATIONS.

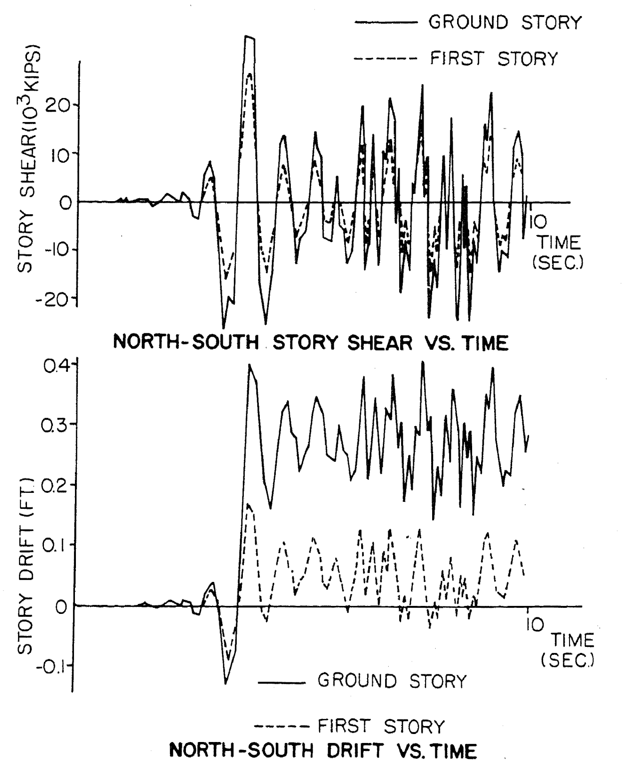

FIG. 4 STORY SHEARS AND DRIFTS, PACOIMA DAM ACCELEROGRAM